RP504

SHEAR TESTS OF REINFORCED BRICK MASONRY BEAMS

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ABSTRACT

Eighteen beams of reinforced brick masonry were tested to determine the resistance of such beams to failure by diagonal tension. The beams were 14 feet long and about 1 foot square in cross section. Beams of three different types of construction were made, an equal number with each of two kinds of brick. A 1:3 Portland cement mortar, with addition of lime equal to 15 per cent of the volume of the cement, was used in all beams. Each beam contained six ½-inch square steel bars as tensile reinforcement. Tensile and shear tests of the bond between mortar and brick and pull-out tests of steel bars embedded in brick masonry were made to supplement the data from the beam tests.

Positions of the neutral axes in the beams varied with kind of brick, arrangement of bricks in the beams, and loads. The ratio of depth to neutral axis to depth of the tensile reinforcement increased with an increase in the number and total thickness of mortar joints in the masonry. The position of the neutral axis corresponded to that calculated by means of the design formulas applying to beams of reinforced concrete, with an assumed modulus of elasticity of the masonry equal to 50 to 70 per cent of that of the masonry piers.

The failures of all beams were accompanied by cracks near the ends of the beams, between a support, and the nearer load. The cracks were evidence of failures by diagonal tension.

Maximum shearing stresses for the different types of beams ranged from 65 to 159 lbs./in.^{2} Resistance to diagonal tension increased with an increase in the proportion of bricks laid with staggered joints. Shearing strengths of the beams were in the same order as shearing and tensile bond strengths of small masonry specimens.

With the relatively absorbent bricks used, tensile and shearing strengths of the masonry were much greater when bricks were wetted before laying than when laid dry.

Bond strengths as determined by pull-out tests of ½-inch square deformed bars embedded about 8 inches in brick masonry ranged from 870 to 950 lbs./in.^{2} Differences in the kinds of brick and curing conditions did not cause significant changes in bond strength.

CONTENTS

I. INTRODUCTION

Brick masonry containing steel reinforcement in the form of bars bands or straps, wires or mesh apparently is one of the oldest forms of reinforced masonry construction, and there already exists a fund of information relating to its structural value.[1] Resistance of brickwork to compressive stresses has been extensively investigated, and the information obtained is available in published reports [2] of tests on piers and walls. Data on resistance of brickwork beams to shearing stresses are not as complete as desired and the chief purpose of tests described herein was to obtain information on resistance of reinforced brickwork beams to diagonal tension failures. The compressive strength of masonry having a like arrangement of the bricks, with reference to the direction of stress to that in the beams was also determined, compressive tests being made on piers resembling short lengths of the beams. Tests of the adhesion of mortar to the bricks and to steel were made to supplement the data from the beam tests.

The tests were made by the Bureau of Standards with the cooperation of the Common Brick Manufacturers Association. The association paid for the materials and labor for the construction of all specimens. The beams were built under supervision of and were tested by members of the bureau staff.

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[1] Edw. E. Krauss and Judson Vogdes, Reinforced Brick Masonry: History, Summary of Tests. Structures Erected and Bibliography to Date, Report No. 5 of Committee on Reinforced Brick Masonry, National Brick Manufacturers Research Foundation, February, 1932. Reinforced Brickwork: A New Construction Material, Engineering News-Record, vol. 109, p. 71, July 21, 1932.

[2] The Building Code Committee of the Department of Commerce has prepared a mimeographed circular giving test data obtained prior to 1926. B. S. RP108, "Compressive Strength of Clay Brick Walls" describes tests of 297 masonry specimens, completed in 1928.

II. DESCRIPTION OF SPECIMENS AND THE TESTING METHODS

1. BRICKS

Bricks from two localities, Chicago and Philadelphia, were used. Both kinds of bricks were made from surface clays and were formed by the stiff-mud and end-cut process. The Chicago bricks were irregular in shape, contained lime nodules, and were considerably laminated. The Philadelphia bricks were somewhat more regular in shape, quite free from lime nodules, but also laminated. In so far as applicable, methods of specification C67–31 of the American Society for Testing Materials [3] were followed in making the tests.

2. MORTAR

The mortar was proportioned by weight to give a mixture approximately equivalent by volume to 1 part of Portland cement to 3 parts of loose, damp sand, with an addition of hydrated lime equal to 15 per cent of the volume of cement. The weights used were 94 pounds of cement and 6 pounds of hydrated lime to 220 pounds of dry sand, water being added in the amounts desired by the masons.

On each day during construction of the beams a sample of mortar was taken from a mason's board and tested for consistency on the 10-inch flow table,[4] and six 2 by 4 inch cylinders were cast from the same mortar. After remaining in the mold for one day three of each set of cylinders were immersed in water at 70° F. and the other three were stored with the beams. All cylinders were tested for compressive strength at the age of 28 days.

3. REINFORCEMENT

The steel reinforcement consisted of deformed ½-inch square bars. Specimens cut from five bars were subjected to tensile tests following the methods (in so far as applicable) of specification A15–14 of the American Society for Testing Materials.[5]

4. BEAMS

(a) TYPES

Eighteen beams were tested, each being 14 feet long and roughly 1 foot square in cross section. Nine beams were built with common bricks from Chicago and the other nine with common bricks from Philadelphia. The specimens were of three different types of construction, there being three beams of each type with each kind of brick. As illustrated in Figure 1, the bricks in beams of type A were laid in common American bond as for a wall 12½ inches in thickness; in those of type B the bonding was similar to that in a lintel beam with exposed soldier courses. Beams of type C were built on end in which position they resembled portions of a 12½-inch wall with all bricks laid as stretchers and having vertical reinforcement near one face as if designed to resist lateral pressures.

Each beam contained six ½-inch square deformed bars as tensile reinforcement. The ends of all bars were bent into hooks in order

[3] 1931 Supplement to Book of A. S. T. M. Standards, p. 5.

[4] The flow tests were made by the method of Federal specification SS-C-181 for cement; masonry (Jan. 6, 1931).

[5] Book of A. S. T. M. Standards, Pt. I, p. 132, 1930.

to minimize the probability of beam failures resulting from slipping of the bars. The piers contained no reinforcement, but each one was similar otherwise to a short length (about 34 inches) of a beam.

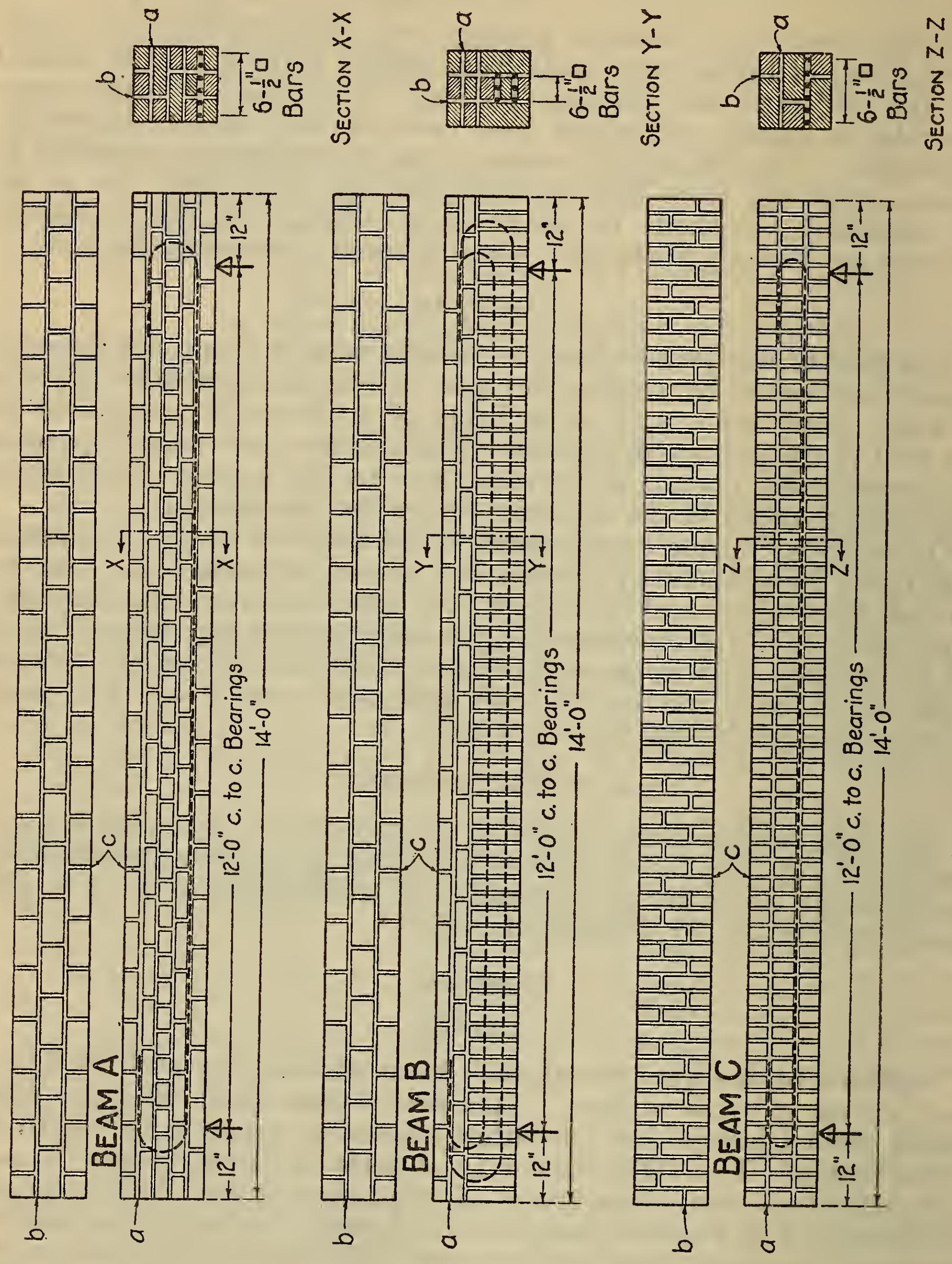

FIGURE 1.—*Details of the test beams*

(b) IDENTIFICATION SYMBOLS

The following symbols, in the order given, are used to identify the beams:

Bricks	C = Chicago. P = Philadelphia.
Type of beam	A as shown in Figure 1. B as shown in Figure 1. C as shown in Figure 1.

Numbers 1, 2, and 3 indicate individual beams.

(c) CONSTRUCTION

The beams were built in the laboratories of the Bureau of Standards by a mason contractor. Bids were obtained from three contractors and the work of constructing the specimens was awarded to the lowest bidder.

The bricks for the beams were dumped from the delivery trucks on an outdoor concrete pavement. On the day before using the bricks they were sprinkled in the pile until the water flowed from every portion. They were again sprinkled in the same manner just before laying.

Beams of types A and B were built on horizontal wooden forms and those of type C were built on end on the laboratory floor. Views of one beam of each type are shown in Figure 2.

The beams were built by two masons working together. The masons were instructed to produce masonry having the spaces between the bricks well filled with mortar, without specifying the method. The method of filling the vertical joints by "slushing" the mortar rather than by "shoving" the bricks was chosen by the masons. During the building of the beams one of the masons was cautioned not to furrow the horizontal beds while the other made smooth spread beds without special effort. Although these masons had no previous experience with reinforced brickwork, the two masons and one helper built the last 12 beams and 12 piers at the rate of one beam and one pier per 3¼ hours.

The average thicknesses of mortar joints as determined from measurements of the beams are given in Table 1.

TABLE 1.—*Thickness of the mortar joints in beams*

Beam No.	Average thickness of mortar joints [1]			Beam No.	Average thickness of mortar joints [1]		
	Horizontal *a*	Vertical in top course			Horizontal *a*	Vertical in top course	
		Longitudinal *b*	Cross *c*			Longitudinal *b*	Cross *c*
	Inches	*Inches*	*Inch*		*Inches*	*Inches*	*Inch*
CA-1	0.66	0.90	0.52	PA-1	0.79	0.85	0.54
CA-2	.66	.90	.69	PA-2	.78	.80	.54
CA-3	.66	.94	.60	PA-3	.59	.70	.54
Average	.66	.91	.60	Average	.72	.78	.54
CB-1	.68	.66	.47	PB-1	.85	.87	.50
CB-2	.70	.68	.53	PB-2	.80	.74	.53
CB-3	.65	.87	.63	PB-3	.80	1.03	.58
Average	.68	.74	.54	Average	.82	.88	.54
CC-1	.99	1.25	.52	PC-1	1.12	1.40	.52
CC-2	1.01	1.25	.44	PC-2	1.02	1.33	.48
CC-3	1.01	1.15	.44	PC-3	1.01	1.13	.48
Average	1.00	1.22	.47	Average	1.08	1.29	.49

[1] With the beam in the same position as during testing. The location of joints *a*, *b*, and *c* are shown in Figure 1.

(d) AGING

The beams of types A and B were lifted from the forms when one week old and were stored, until tested, in stacks which allowed free circulation of air around each specimen. Those of type C were placed in a horizontal position in the same stacks. All beams were tested at ages ranging from 27 to 29 days.

(e) METHOD OF TEST

The beams were tested in a vertical screw-testing machine having a capacity of 600,000 pounds. They were supported over a span of 12 feet and loaded along two lines, each 3 feet from mid span. The type of supports and method of transmitting the loads to the beam are illustrated in Figure 3.

Measurements were made of deflections at mid span of the beams and of deformations in the masonry and steel along longitudinal gage lines near mid span. Deflections were measured by means of micrometer dials which indicated the vertical displacement of the upper surface of the beam at mid span relative to a metal frame supported at each end on steel spheres directly over the beam supports. (Shown in fig. 3.) Compressive longitudinal deformations in the upper surfaces of the beams were measured on 20-inch gage lengths by means of a hand strain gage. Tensile longitudinal deformations in the steel over 8-inch gage lengths were measured in one of the bars near each lateral face of the beams by means of gages clamped to the under sides of the bars. Small strap-iron bars, set in vertical joints, were used to support micrometer dial gages and spacing bars for making measurements over longer gage lengths. With this apparatus the deformations in gage lengths of about 40 inches were measured at each lateral face about 1 inch below the upper surface and about 1 inch above the lower surface of a beam.

In making a test the load was increased by increments until deformations indicated stresses in the steel of about 15,000 lbs./in.^{2}. The total load on the beam was then reduced to 1,000 pounds, and finally the load was increased by increments until failure occurred. Usually the devices for measuring strains were removed before the beams failed.

5. PIERS

Eighteen piers, corresponding to the 18 beams, were also built. The piers contained no reinforcement, but each was similar otherwise to a short length (about 34 inches) of a beam and was built in a manner similar to that of the corresponding beam, identified by the same symbol and aged under the same conditions. A view of an unfinished pier is shown in Figure 2.

The piers were tested in compression under central loading in a 10,000,000-pound-capacity testing machine.[6] The lower bearing surface of the pier was bedded in plaster of Paris (calcined gypsum) on the lower platen of the machine. After tilting this platen until the upper surface of the pier was parallel with the upper platen, the upper surface was capped with plaster of Paris.

Vertical compressometers having gage lengths usually between 20 and 30 inches were attached to two opposite sides of the pier. Read-

[6] Described in Bureau of Standards Research Paper No. 108, p. 526.

Figure 2.—*Test beams under construction*

Note the type C beam on end at the left.

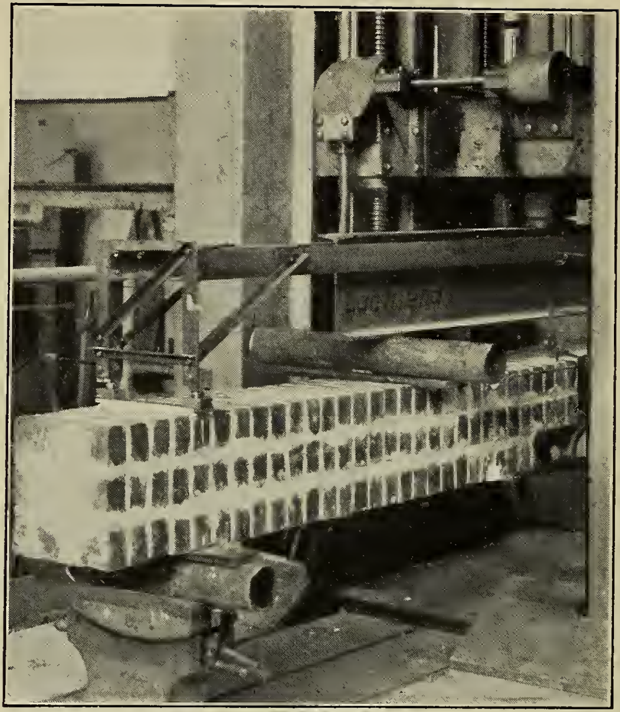

FIGURE 3.—*Beam PC–3 in testing machine*

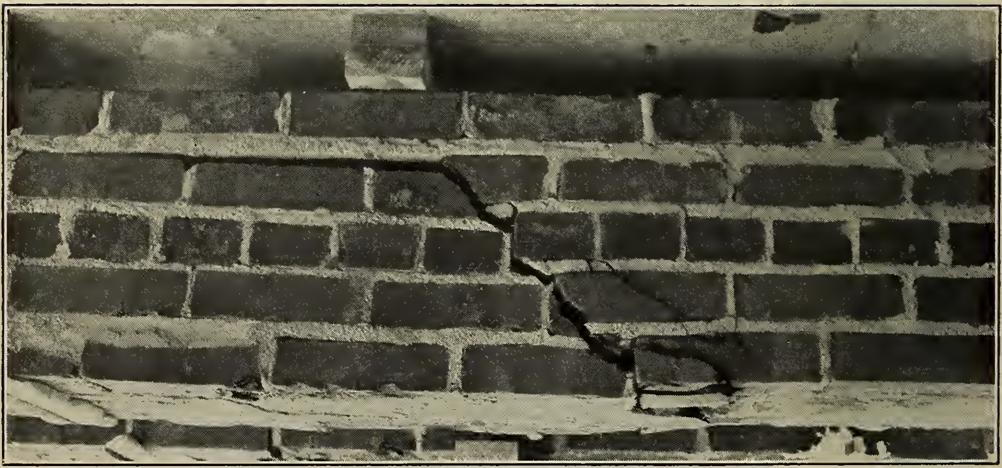

FIGURE 7.—*Beam PA–3 after diagonal tension failure*

ings were taken at equal increments of load usually obtaining about eight readings as the stress increased from 0 to 1,000 lbs./in.^{2}, the load being held constant while each set of readings was made. The compressometers were then removed and the machine run at constant speed until failure occurred.

6. BRICK-MORTAR TENSILE SPECIMENS

As the tensile strength of masonry in beams without web reinforcement is the chief source of resistance to diagonal tension, the factors which affect tensile strengths of brick and mortar specimens probably are the same as those governing web resistance of beams. Failures of specimens of brick and mortar when subjected to tensile stresses might conceivably occur by failures of the bricks, of the mortar, of the adhesion between mortar and brick, or of any combination of these. The purpose of tests described was to compare tensile strengths of specimens made with the same types of bricks and the same mortar as were used in the beams. Variations in moisture content of the bricks when laid and in curing conditions were included in the study in order to obtain an estimate of their influence on the strengths of the masonry.

Tests of strength of bond in tension of brick-mortar joints were made. The specimens consisted of two bricks laid flatwise, separated by mortar, the cross section of the joint being 30 square inches for standard size bricks. Twenty test specimens were made for each of the following conditions: Two makes of bricks (Chicago and Philadelphia), dry bricks and dry cure, dry bricks and damp cure, wet bricks and dry cure, and wet bricks and damp cure.

Bricks were considered as dry after one week's storage in a steam heated room. Some bricks were used after 48 hours in a drying oven at 220° F., followed by 24 hours' cooling in the laboratory.

For wetting bricks, the procedure was to totally immerse previously dried bricks for one hour, stand on end in air for one-half hour, and then make the test specimens within the next half hour.

All construction was done in a room kept at a temperature of 70° ±1° F. and a relative humidity of 40 to 60 per cent. The "dry cure" specimens were left in this room for 48 hours after construction and then removed to a laboratory at "room temperature." The "damp cure" specimens were removed at the end of the half hour construction period to the damp storage room of the concrete laboratory, which is kept at a temperature of 70° ±1° F. and a relative humidity of over 90 per cent.

The mortar used was a cement-lime-sand mixture of the same proportions as that used in constructing the beams. An attempt was made to have the flow immediately after mixing between 110 and 120 per cent. The mortar was proportioned and mixed dry. Time was counted from the moment of adding the water to the mortar mix. A flow test was made before starting construction of the test specimens. Mortar was thrown on the flat of the bottom brick. The top brick was quickly put in place with a shoving motion, using considerable pressure. Excess mortar was cut off with a trowel. Since comparison of the two makes of bricks was considered the main purpose of the investigation, the individual specimens were constructed alternately of Chicago and Philadelphia bricks.

The action of the dry bricks of both makes was to suck water out of the mortar, hence the utmost possible speed was used in getting the bricks in place. At best, a number of joints in the dry brick specimens were imperfectly filled.

Care was taken to avoid jarring specimens after construction. In spite of care used in handling, a considerable number of the dry brick specimens separated before testing.

It was the intention to make all tests at 28 days, hence the specimens were removed from damp storage at the end of 24 days, exposed in the laboratory for two days and then capped. Circumstances required that some of the tests be delayed, but the damp cure was restricted in all cases to 24 days.

The method of determining the tensile strength was essentially that used for testing whole bricks in tension described in another paper.[7] Palmer and Hall[8] further describe the apparatus and method.

7. BRICK-MORTAR SHEARING SPECIMENS

The shearing specimens were equal in number and made with the same mortar mixture and with the same procedure as the brick mortar tensile specimens. The specimens for shear were made by laying three bricks flat, the top and bottom bricks having their ends in line while the center brick was displaced lengthwise from one-half to three-quarters of an inch.

The projecting ends of the two outer bricks of each specimen were capped with plaster of Paris, the surfaces of both caps being in one plane. The projecting end of the center brick was also capped, its surface being as far as possible parallel to the surfaces of the two other caps.

The specimens after capping were loaded in compression, the load being applied to the center brick and the specimen resting on the ends of the two outside brick. This is the method of Douty and Gibson.[9]

8. PULL-OUT TESTS

Specimens for pull-out tests consisted of a ½-inch square deformed steel bar imbedded lengthwise in a mortar joint of a small brick pier. The same variables in brick and curing were used as for the brick-mortar tensile and shearing specimens, but the brick were wetted before laying. A deformed steel bar (one-half inch square) was held vertically by clamps, the lower end resting on oiled paper. Around this bar was built a small brick pier approximately 8 by 8 inches in cross section and three bricks high. The middle course of bricks were laid as headers with respect to the top and bottom courses. Care was taken to secure imbedding of the bar in mortar and to avoid contact of brick with the bar. All joints were filled with mortar. No dry bricks were used in this series, but both kinds of cure were employed. The mortar was the same as that used for the tensile and shearing specimens.

Before testing, the specimens were capped with plaster of Paris on top of the bricks, care being taken to have the capped surface smooth and normal to the projecting steel bar.

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[7] J. W. McBurney, Strength of Brick in Tension, J. Am. Ceramic Soc., vol. 11 (2), pp. 114–117, 1928.

[8] L. A. Palmer and J. V. Hall, Durability and Strength of Bond Between Mortar and Brick, B. S. Jour. Research vol. 6 (3), pp. 473–492, 1931.

[9] R. F. Douty and H. C. Gibson, Influence of the Absorptive Capacity of Brick Upon the Adhesion of Mortar, Proc. Am. Soc. Testing Materials, vol. 8, pp. 513–530, 1908.

During a test the specimen was inverted, resting on a steel plate supported by the top head of the testing machine. The bar projected downward through a ¾-inch hole in the steel plate and was gripped by wedges in the movable head of the testing machine.

III. RESULTS OF THE AUXILIARY TESTS

1. BRICKS

Results of tests of the bricks are given in Table 2, in which each average value is the mean from 25 tests. As a measure of the dispersion of the results of single tests about their corresponding averages, there are given in Table 2 values for the standard deviations, which were calculated by means of the following formula:

$$\sigma=\sqrt{\frac{\Sigma v^2}{n-2}}$$

where

σ = standard deviation.

n = number of individual values.

Σv^2 = sum of the squares of the deviations of the single values from their mean.

The values of standard deviations given in other tables were calculated by means of the same formula.

2. MORTAR IN THE BEAMS

Results of tests of the specimens representing the mortar in the beams are given in Table 3.

TABLE 2.—*Properties of the bricks*

Each value was derived from results of tests of 25 bricks

Property	Kind of bricks			
	Chicago		Philadelphia	
	Mean value	Standard deviation	Mean value	Standard deviation
Length ... inches	8.00	0.08	8.15	0.14
Width ... do	3.64	.06	3.72	.06
Thickness ... do	2.23	.03	2.29	.02
Compressive strength flatwise ... lbs./in.²	3,910	860	4,510	840
Compressive strength edgewise ... do	4,280	950	5,240	950
Compressive strength endwise ... do	7,030	1,330	4,200	1,750
Modulus of rupture flatwise ... do	1,530	570	650	340
Absorption by 5 hours immersion ... per cent	8.8	2.7	11.1	1.6
Absorption by 48 hours immersion ... do	10.8	2.8	12.7	1.9
Absorption by 5 hours boilding ... do	14.7	2.6	16.1	1.8

TABLE 3.—*Properties of mortar for the beams*

Each value was derived from results of 21 tests

Property	Mean value	Standard deviation
Flow ... per cent	108	8.2
Compressive strengths of 2 by 4 inch cylinders, age 28 days:		
Dry storage ... lbs./in.²	2,340	440
Damp storage ... do	3,740	530

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3. REINFORCEMENT

Tensile properties of the ½-inch square deformed steel bars are given in Table 4.

Table 4.—*Properties of reinforcement for the beams*

Each value was derived from the results of five tests

Property	Mean value	Standard deviation
Cross sectional area .... inches ²	0. 240	0. 002
Proportional limit .... lbs./in.²	45, 200	6, 700
Yield point .... do	51, 100	1, 600
Tensile strength .... do	81, 600	1, 400
Modulus of elasticity .... do	29, 400, 000	1, 100, 000
Elongation in 8 inches .... per cent	22. 4	2. 0

4. BRICK-MORTAR TENSILE SPECIMENS

Results of tests of the brick-mortar tensile specimens are given in Table 6.

Table 5.—*Properties of mortar for the tensile and shear test specimens*

Each value was derived from results of from 40 to 51 tests. Strength tests at ages ranging from 28 to 60 days

Property	Mean value	Standard deviation
Initial flow .... per cent	116	9. 8
Flow one-half hour after mixing .... do	92	14. 1
Tensile strengths of briquettes with:		
Dry storage .... lbs./in.²	290	25
Damp storage .... do	470	34
Compressive strengths of 2 by 4 inch cylinders with:		
Dry storage .... lbs./in.²	3, 410	400
Damp storage .... do	4, 010	250

Table 6.—*Results of tensile tests of bond between mortar and brick*

Age at test 28 to 60 days

Brick		Storage	Number of specimens	Average strength	Standard deviation
Kind	Condition when laid				
				Lbs./in.²	*Lbs./in.²*
Chicago	Dry	Dry	16	38	17
		Damp	16	38	13
	Wet	Dry	20	55	17
		Damp	20	61	16
Philadelphia	Dry	Dry	16	18	10
		Damp	17	27	12
	Wet	Dry	20	53	18
		Damp	20	44	13

Mortar used in brick-mortar tensile specimens and in brick-mortar shearing specimens had the same proportions of dry constituents as the mortar in the beams and piers. In the latter, water was added as desired by the masons; while in the brick-mortar specimens, made under more careful laboratory conditions, the amount of water to give

a flow of approximately 110 per cent was used. For this reason, properties of the mortar, given in Table 5, for brick-mortar specimens differ somewhat from values of Table 3 for the mortar of the beams.

Where test specimens represented wetted bricks, the characteristic failure was not at the junction of brick and mortar but was a failure in the brick. Chicago bricks left a "skin" adhering to the mortar, and Philadelphia bricks frequently pulled off or sheared off their flats to a depth of one-eighth of an inch. In other words, when bricks had been wetted, bond between brick and mortar exceeded the strength of the brick. On the other hand, a few of the "dry brick" specimens showed separation in the mortar, mortar adhering to both bricks. The difference between number of tests indicated and the 20 test specimens originally constructed represents failures of bond occurring by handling the specimens. The number of tests is too few and the variation too great to permit much weight to be given either to averages or distributions but several conclusions are evident: First, wetting Chicago and Philadelphia bricks much increased strength of bond; second, the Chicago brick tended to give stronger bonds than the Philadelphia brick.

5. BRICK-MORTAR SHEARING SPECIMENS

Results of tests of brick-mortar shearing specimens are given in Table 7. In general, results of these tests are similar to the results of tests of the brick-mortar tensile specimens.

TABLE 7.—*Results of shear tests of bond between mortar and brick*

Age at test, 28 to 60 days

Brick		Storage	Number of specimens	Average strength	Standard deviation
Kind	Condition when laid				
				Lbs./in.²	*Lbs./in.²*
Chicago	Dry	Dry	16	100	50
		Damp	12	115	47
	Wet	Dry	20	275	59
		Damp	19	245	101
Philadelphia	Dry	Dry	10	91	35
		Damp	11	120	38
	Wet	Dry	19	231	86
		Damp	20	173	70

6. PULL-OUT TESTS

Results of the pull-out tests are given in Table 8. As shown by the data of this table there was not a significant difference in bond strength of specimens made with Chicago or Philadelphia bricks. Furthermore, curing conditions did not affect results significantly. Failure was by splitting of the brickwork usually into halves but in some cases into quarters. Specimens giving highest individual loads had strengths which exceeded the proportional limit of the steel.

TABLE 8.—*Results of pull-out tests*

Specimens were deformed steel bars, one-half inch square, embedded about 8 inches in 8 by 8 inch square brick masonry piers

Brick		Storage	Number of specimens	Age at test	Average bond strength	Standard deviation
Kind	Condition when laid					
				Days	*Lbs./in.²*	*Lbs./in.²*
Chicago	Wet	Dry	5	32–48	880	140
Philadelphia	do	do	4	32–48	950	140
Chicago	do	Damp	5	41–57	870	190
Philadelphia	do	do	5	41–57	920	150

The maximum bond stresses were slightly greater than those reported by Abrams [10] for deformed bars in 1:2:4 concrete two years old. It has been shown that for smooth bars embedded in concrete the bond strength is more dependent on length of embedment than on other size and shape factors.[11] The lengths of embedment in these two series were about the same (8 inches). Hence, if the effects of size and shape of specimens are approximately the same with deformed bars as with smooth bars, it may be concluded that bond strengths in the brick masonry specimens were about the same as in 1:2:4 concrete specimens tested by Abrams.

7. PIERS

Results of tests of the piers are given in Table 9. There was not a marked departure from a linear relation between loads on the piers and their compressive deformations until 25 per cent of the maximum load was reached, after which there was a tendency for the deformations to increase more rapidly than the loads increased.

Table 9 gives values of the secant modulus of elasticity at a stress of 250 lbs./in.², obtained by dividing this stress by the corresponding compressive strain. Values of the secant modulus of elasticity for any stresses less than one-fourth of the maximum did not differ significantly from those given.

TABLE 9.—*Results of pier tests*

Pier	Secant modulus of elasticity to 250 lbs./in.²	Compressive strength	Pier	Secant modulus of elasticity to 250 lbs./in.²	Compressive strength
	Lbs./in.²	*Lbs./in.²*		*Lbs./in.²*	*Lbs./in.²*
CA-1	2,690,000	1,735	PA-1	1,050,000	1,438
CA-2	2,400,000	1,955	PA-2	1,800,000	1,577
CA-3	2,340,000	1,879	PA-3	1,770,000	2,039
Average	2,480,000	1,856	Average	1,540,000	1,688
CB-1	2,120,000	1,956	PB-1	980,000	1,210
CB-2	2,140,000	2,458	PB-2	1,020,000	1,421
CB-3	2,380,000	1,661	PB-3	1,220,000	1,535
Average	2,210,000	2,025	Average	1,070,000	1,389
CC-1	1,570,000	1,235	PC-1	740,000	1,072
CC-2	960,000	810	PC-2	1,070,000	1,423
CC-3	1,020,000	1,019	PC-3	800,000	1,112
Average	1,180,000	1,021	Average	870,000	1,202

[10] Duff A. Abrams, Tests of Bond Between Concrete and Steel, Bul. No. 71, University of Illinois.
[11] W. H. Glanville, Studies in Reinforced Concrete. I. Bond Resistance, Building Research Tech. Paper No. 10, Dept. of Sci. and Ind. Res., London, 1930.

Both the moduli of elasticity and compressive strengths of the piers were greater for piers of types A and B, having some bricks on end, than for those of type C having all bricks flatwise. As indicated by the data of Table 9 on compressive strength of the piers and that of Table 2 on physical properties of the bricks, there were not large differences in strength properties of either bricks or piers.

IV. RESULTS OF BEAM TESTS

1. DEFORMATIONS IN THE BEAMS

Load deformation curves for the brickwork and the steel and load-deflection curves for the beams are shown in Figures 4 and 5. Values shown for the strains in the brickwork and steel are averages calculated from data obtained with the 40-inch gage length extensometer attached to the sides of beams and the readings of the strain gages. The curves are similar in shape to those representing like data from tests of concrete beams containing heavy reinforcement. Disregarding the portion due to the partial release of the load, the curves tend to consist of a short straight portion, corresponding to the first stage of loading, while the brickwork was still effective in resisting tensile stresses; then a curved portion as the failure of the brickwork in tension took place and then a nearly straight portion. Usually this was followed by another curved portion similar to that of a stress strain diagram representing the later stages in a compressive test of brick masonry.

Some bricks under the reinforcement bars at mid span were removed, prior to testing the beams, in order to facilitate attachment of strain gages on the bars. Due partly to their removal and partly to lack of sensitivity of the long and the short gage tensometers, reliable values for extensibility of the brickwork were not obtained.

When loads on the beams were reduced to 1,000 pounds after the first loading, strain and deflection indicators did not return to the positions taken under the first application of a load of 1,000 pounds. The differences, which are measures of sets produced by the loadings, were equal usually to from one-seventh to one-fifth of the deformations under the greatest loading which preceded.

2. POSITIONS OF THE NEUTRAL AXIS

Average positions of the neutral axes for each type of beam at several loads as determined from measured deformations are indicated in Figure 6, k being the ratio of depth of neutral axis to depth of centroid of the steel. Each plotted point represents a value obtained by averaging the data for three like beams. On the same diagram are shown horizontal lines indicating positions of the neutral axes corresponding to several values of n, calculated by means of the following well known formula:

$$k=\sqrt{2pn+(pn)^2}-pn$$

where k = ratio of depth to neutral axis to depth of the centroid of the tensile reinforcement.

p = ratio of area of tensile reinforcement to area of masonry above the centroid of the tensile reinforcement.

$n=\frac{E_s}{E_m}$ = ratio of the modulus of elasticity of steel to that of the masonry.

As loads were increased there was a tendency for the neutral axis to rise in the beams of types A and B, indicating a gradual lessening of tensile resistance of the brickwork. The tendency of the neutral

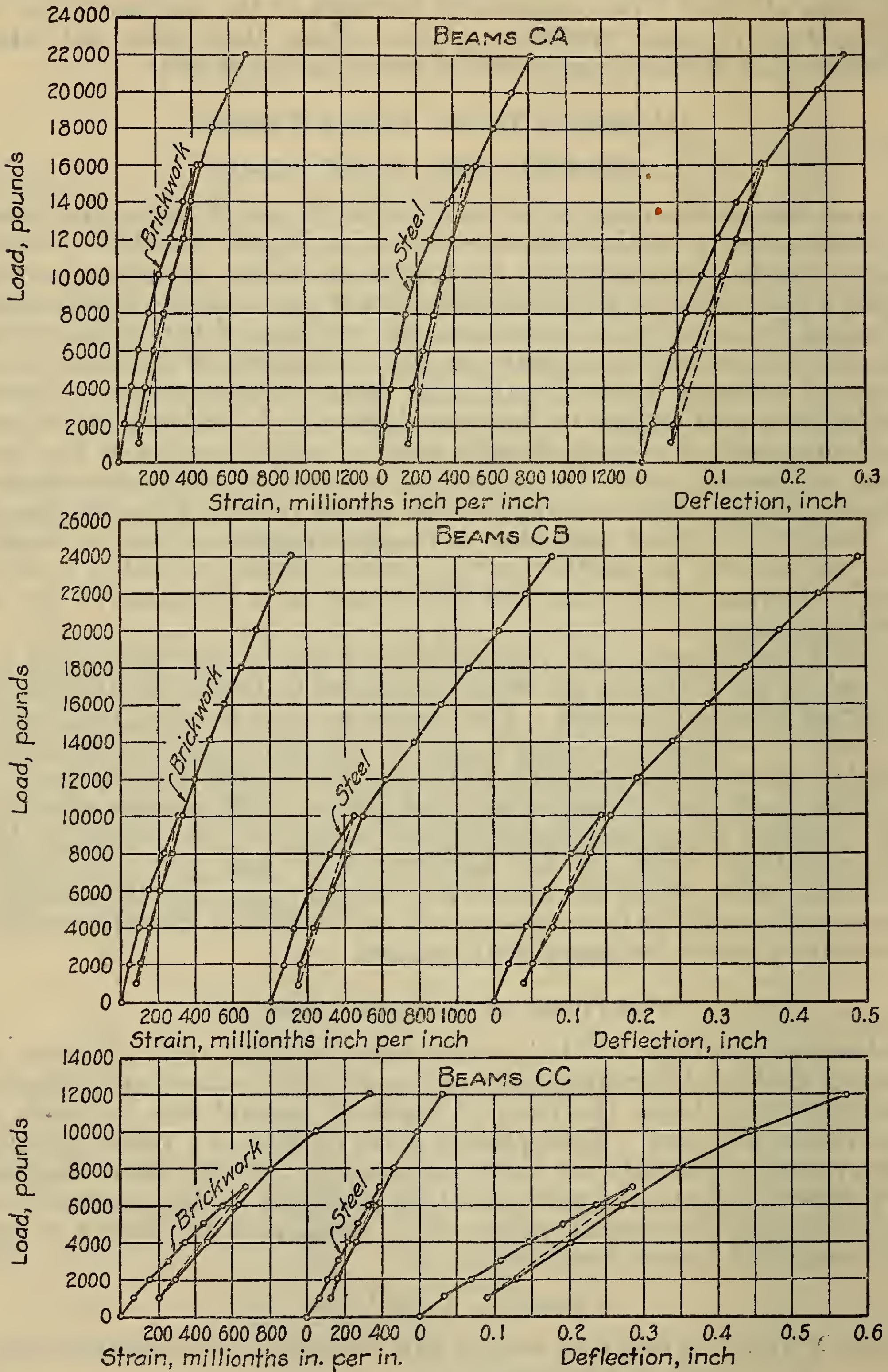

Figure 4.—*Load-deformation curves for beams with Chicago brick*

axis to rise continued until the compressive stresses in the brickwork were so great that the rate of change of strain with compressive stress in the brickwork had become great enough to either partially or com-

pletely overcome this tendency. Changes in the depths to the neutral axis with increased loads on these beams resembled those in reinforced concrete beams of similar shape and reinforcement, except that the reduction of tensile resistance of the brickwork in beams of types A and B seemed to take place more gradually than is common in beams of concrete. As the modulus of elasticity of the masonry E_m varies with stress and as the effects of tensile resistance of the

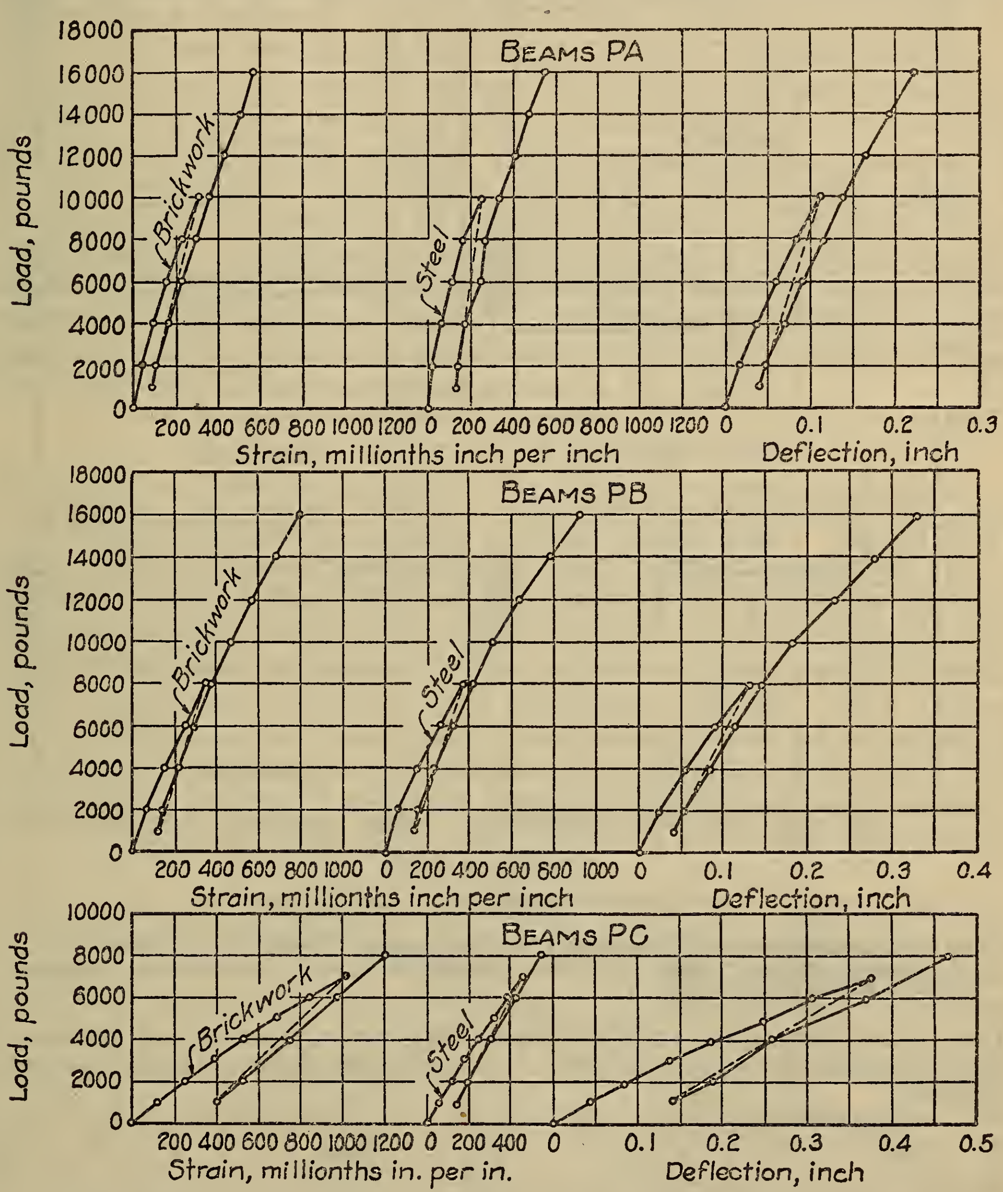

FIGURE 5.—*Load-deformation curves for beams with Philadelphia brick*

masonry are neglected in the derivation of the design formulas, it is desirable to determine what values of n correspond in the formula $k=\sqrt{2pn+(pn)^2}-pn$ to the observed values of k. For this purpose it is best to use data obtained while conditions in the beams most nearly correspond to those assumed in the derivation of the design formulas. The minimum values of the ratio k usually were obtained when loads on the beams were sufficiently large to have caused tensile cracks in the masonry below the neutral axis and yet not large enough to have produced a rapid plastic yielding of the masonry in com-

pression. The minimum values of k observed for two or more loads are given in Table 10, together with values of n calculated by means of the foregoing formula.

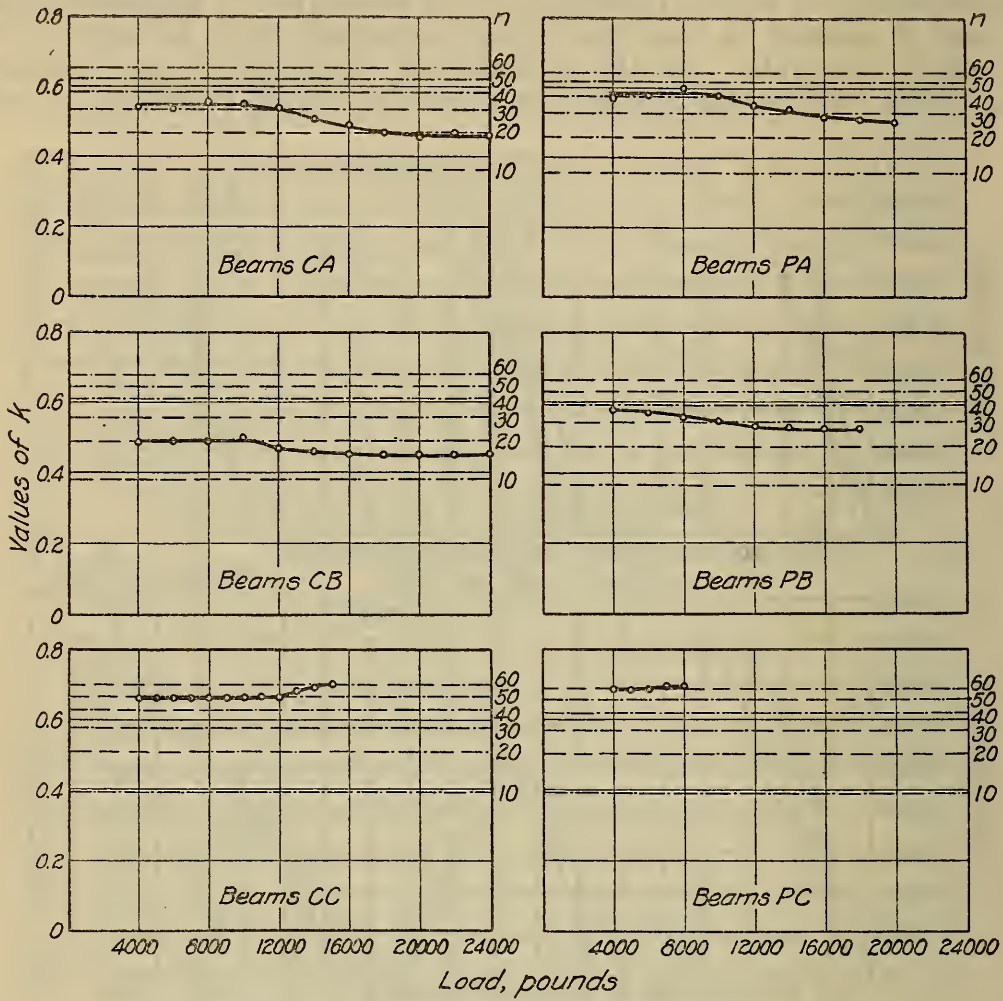

FIGURE 6.—*Position of the neutral axes*

The circles indicate values of k determined from the measured deformations. Calculated values of k corresponding to the values of n, shown on the right-hand scale of each graph, are indicated by the horizonal dash-dot lines

TABLE 10.—*Values of n calculated from dimensions of beams and positions of neutral axes during tests. Values of k were determined from the long gage deformations*

Beam	p	k	n	Beam	p	k	n
	Per cent				*Per cent*		
CA-1	1.03	0.43	16	PA-1	0.97	0.54	33
CA-2	1.01	.44	17	PA-2	.97	.48	23
CA-3	1.00	.45	18	PA-3	1.02	.53	29
CB-1	1.20	.44	15	PB-1	1.07	.48	21
CB-2	1.17	.45	16	PB-2	1.10	.57	34
CB-3	1.16	.45	16	PB-3	1.05	.56	34
CC-1	1.31	.66	50	PC-1	1.24	.70	66
CC-2	1.31	.62	38	PC-2	1.25	.69	62
CC-3	1.30	.68	56	PC-3	1.27	.69	60

These calculated values of n were greater for beams of Philadelphia bricks than with those of Chicago bricks. They were greatest with

beams of type C and least with those of types A and B, the values corresponding in order of magnitude to the average number and total thickness of joints per unit length of beam in the upper two courses of bricks. However, no general relations between dimensions of the mortar joints and values of n exist, probably on account of the laminar structure of the bricks.

Secant moduli of elasticity of the piers were in all cases larger than the effective moduli of the masonry of beams as indicated by values of n in Table 10. The ratios of these values range from 0.5 to 0.9. A close correspondence would not be expected because of difference in distribution of stresses over their unsymmetrical cross sections. The entire section of piers A and C resembled more nearly than those of type B the portion of the masonry in the beams which were in compression. For these two types the average ratios of effective moduli of masonry in beams to moduli of the piers ranged from 0.5 to 0.7. Fortunately, from the standpoint of design, accurate values of n are rarely required for satisfactory results, as an error of 50 per cent in the assumed value of n would rarely cause an error of more than 15 per cent in the calculated stress in the masonry or more than 4 per cent in the calculated stress in the steel.

3. TYPES OF FAILURES

Failures of all beams were accompanied by cracks near the ends of the beams between a support and the nearer load. Figure 7 is a view of two beams after testing. These cracks tended to extend diagonally upward from the support toward the load line, in some instances passing through the bricks for a part of their lengths while in others following the mortar joints entirely. They usually became visible before the maximum load had been applied. In a typical case, the appearance of the crack was accompanied by a falling off of load and a rather abrupt increase in deflection. After this, with the machine running at constant speed, load increased more slowly, but the maximum loads were usually from 5 to 10 per cent greater than when the crack was first observed.

All beams except CA–2 and CC–1 failed by diagonal tension. The tensile reinforcement in beam CA–2 began to yield before the maximum load had been applied. Using the ordinary formulas for working stresses in concrete beams and considering the position of the neutral axis as observed, the calculated maximum stresses under maximum load in this beam were 54,000 lbs./in.^{2} in the steel and 2,400 lbs./in.^{2} in the masonry. Strain-gage readings indicated yielding of masonry in the upper surface of beam CC–1 and spalling was observed prior to the maximum load. Calculated stresses under the maximum load were 34,000 lbs./in.^{2} in the steel and 1,340 lbs./in.^{2} in the masonry. The beams were so conservatively designed against failures by slipping of the bars that the bond stresses developed in the beam tests have no significance.

4. RESISTANCE TO DIAGONAL TENSION

(a) EFFECT OF ARRANGEMENT AND BONDING OF BRICKS IN BEAMS

Resistance of the beams to failures by diagonal tension was affected markedly by the bonding of the bricks as is shown by the data of Table 11. With the Chicago bricks the ratios of the average maxi-

mum shearing stress to that for type C beams were, respectively, 1.69, 1.38, and 1.00 for beams of types A, B, and C; corresponding ratios for beams of Philadelphia bricks were 1.49, 1.42, and 1.00.

TABLE 11.—*Results of beam tests*

Beam No.	Width b	Depth d	j	Maximum load W	Maximum shearing stress $v=\frac{Vr}{bjd}$
	Inches	*Inches*		*Pounds*	*Lbs./in.*²
CA-1	12.73	11.00	0.85	36,700	154
CA-2	12.73	11.25	.85	41,750	171
CA-3	12.80	11.30	.85	37,600	153
Average	12.75	11.18	.85	38,680	159
CB-1	12.23	9.84	.85	28,500	140
CB-2	12.57	9.84	.85	26,000	123
CB-3	12.67	9.84	.85	27,600	130
Average	12.49	9.84	.85	27,370	131
CC-1	12.90	8.50	.78	18,000	105
CC-2	12.90	8.50	.78	14,900	87
CC-3	12.80	8.65	.78	15,900	91
Average	12.87	8.55	.78	16,270	94
PA-1	12.87	11.95	.83	23,450	92
PA-2	12.77	11.65	.83	22,100	89
PA-3	12.57	11.20	.83	26,000	111
Average	12.74	11.60	.83	23,850	97
PB-1	12.90	10.40	.83	20,000	90
PB-2	12.63	10.40	.83	17,400	80
PB-3	13.23	10.40	.83	24,000	105
Average	12.92	10.40	.83	20,470	92
PC-1	13.27	8.75	.77	8,650	48
PC-2	13.20	8.75	.77	14,400	81
PC-3	13.00	8.75	.77	11,700	67
Average	13.16	8.75	.77	11,580	65

NOTE.—$j=1-\frac{k}{3}$, where k is the average of experimentally determined values of ratio of depth of neutral axis to effective depth. The values of k used were taken from Figure 6.

Maximum shearing stresses were in the same order as the proportion of bricks laid with staggered vertical joints. As shown in Figure 1, joints which were vertical in the type C beams during a test were not staggered; they extended over the full width and from the lower to upper surfaces of the beams. In type B beams all vertical joints were staggered, but those between the soldier bricks of the outer wythes had an unbroken vertical length about equal to the height of three courses of bricks laid flatwise. The vertical joints in the A beams were broken at each course. The proportion of the staggered joints in the different beams my be expressed by approximate numerical values as:

15 out of 15 or 100 per cent in the A beams.
9 out of 15 or 60 per cent in the B beams.
0 out of 9 or 0 per cent in the C beams.

(b) EFFECT OF STRENGTH AND ABSORPTION OF THE BRICKS

Resistance of beams to shearing stresses (Table 11) were in reverse order to compressive strengths flatwise and edgewise of the bricks (Table 2), the strengths of Chicago bricks in these tests being less than that of Philadelphia bricks. Chicago bricks were stronger endwise and when laid as in the top course of beams A and B were subjected to compressive stresses in the direction of greatest strength. However, strengths of the C beams were not in the same order as compressive strengths of bricks in the direction of the axis of the beams. It appears, therefore, that there was no consistent relation between compressive strength of bricks and shearing resistance of masonry.

Shearing resistance of the beams was in the same order as moduli of rupture of the bricks. The moduli of the bricks were determined, however, only in the direction of their lengths. Bricks from both sources were laminated and their moduli in other directions may not have been in the same order. Therefore, the data are not conclusive as to the effect of the moduli of rupture of the bricks on shearing strength of the masonry.

Absorption properties of the two kinds of bricks did not differ greatly, and it does not seem likely that the small differences in absorptions had an important effect on the relative strengths of beams made with them.

(c) EFFECT OF BOND STRENGTH BETWEEN MORTAR AND BRICKS

As noted in the description of tensile and shearing tests of brick-mortar specimens, some specimens were made with wet and the rest with dry bricks. Some were aged in damp storage and others in dry storage. Bricks for the beams were wetted before use. Although the beams were aged in the laboratory where some drying would take place, the rate of drying was probably much less than for the smaller bond test specimens stored in air. Hence, it would be expected that the moisture content during fabrication and storage of the beams was not the same as for any of the bond-test specimens but was intermediate between the dry and wet storage bond-test specimens made with wet bricks. Moreover, as conditions during the first few days of storage have a greater effect upon the properties of Portland cement mortars than those during a similar equal period, it seems likely that the properties of the mortars in the beams were more nearly like those of the damp-cured bond specimens.

Table 12 gives ratios of the average maximum shearing stress of beams of Chicago bricks to those of Philadelphia bricks for each type of beam. These were calculated from the data of Table 11. Similar ratios calculated from data of Tables 6 and 7 for bond tests are given also. The close agreement between ratios for the beams and for the damp-cured bond-test specimens indicates, as would be expected,[12] that tensile and shearing strengths of the masonry were closely related to the resistance to diagonal tension of the masonry beams.

[12] Because the appearance of the cracks indicated failures due to diagonal tension.

TABLE 12.—*Effect of tensile and shearing strengths of masonry on shearing resistance of beams*

[Ratio of strength with Chicago bricks to strength with Philadelphia bricks]

Maximum shearing stress beams:	
A	1.64
B	1.43
C	1.45
Bond strength in tension, wet bricks, damp storage	1.39
Bond strength in shear, wet bricks, damp storage	1.41
Bond strength in tension, wet bricks, air storage	1.04
Bond strength in shear, wet bricks, air storage	1.19

V. CONCLUSIONS

1. Maximum shearing stresses for the different types of beams, which are taken as measures of their resistances to diagonal tension, ranged from 65 to 159 lbs./in.^{2}.

2. Resistance to diagonal tension increased with an increase in the proportion of the bricks laid with staggered joints. Maximum shearing stresses in the beams having all bricks laid flatwise with staggered joints were about 60 per cent greater than in beams with continuous (not staggered) joints.

3. Maximum shearing stresses in beams with one kind of brick were from 40 to 60 per cent greater than in beams made with the other kind. These stresses were in the same order as the tensile and shearing strengths of small masonry specimens made with the same kinds of brick and mortar. The shearing strength of the beams appeared to be independent of the compressive strengths of the bricks. The only qualities of the bricks which appeared to have a major influence were those affecting tensile and shearing strength of the masonry.

4. With the rapidly absorbing bricks used in these tests tensile and shearing strengths of the masonry were much greater when the bricks were wetted before laying than when laid dry. Curing conditions had a relatively small effect; specimens made with dry brick were made stronger by damp curing, while those made with wet bricks were stronger when air cured.

5. The neutral axes in the beams rose during the early stages of the loading, but during the later stages usually became lower. The positions as indicated by minimum observed values of k corresponded to values of $n=\frac{E_s}{E_m}$ ranging from 15 to 66. The effective n varied with the kind of bricks in the beam and increased with an increase in the average number and total thickness of mortar joints in the upper course per unit of length of beam. The effective E_m in the beams ranged from 0.5 to 0.7 of the secant modulus of elasticity of the piers, when the bonding of the bricks in the piers was similar to that in the upper half of the beams.

6. The bond strength at an age of from one to two months as determined by pull-out tests of ½-inch-square deformed bars embedded 8 inches in brickwork ranged from 870 to 950 lbs./in.^{2}. Differences in the kinds of bricks and of curing conditions did not cause significant changes in these bond strengths.

WASHINGTON, September 17, 1932.